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THE SEISMIC CATEGORY I STRUCTURES PROGRAM

by

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ABSTRACT

With the use of different size scale models, the Seismic Category I Structures Program has demonstrated consistent results for measured values of stiffness at working loads. Furthermore, the values are well below the theoretical stiffnesses calculated from an uncracked strength-of-materials approach. The scale model structures, which are also models of each other, have demonstrated scalability between models. The current effort is to demonstrate that the use of microconcrete and other modeling effects do not introduce significant distortions that could drastically change conclusions regarding prototype behavior for these very stiff, shear-dominated structures. Working closely with the technical review group (TRG) for this program, structures have been designed and tests have been planned that will help to resolve issues surrounding the use of microconcrete scale models.

INTRODUCTION

The Seismic Category I Structures Program is being carried out at the Los Alamos National Laboratory under sponsorship of the U.S. NRC, Office of Nuclear Regulatory Research, and has the objective of investigating the structural dynamic response of Seismic Category I reinforced concrete sturctures (exclusive of containment) that are subjected to seismic loads beyond their design basis.

Specific program objectives are as follows:

1. Develop experimental data for determining the sensitivity of structural behavior (acceleration, displacement, frequency, structural stiffness, etc.), in the elastic and inelastic ranges, of noncontainment Category I structures to variations in configuration and earthquake loading.

2. Identify the sensitivity of floor response spectra changes to the variations selected in No. 1.

3. Develop a way of representing damping in the inelastic range. Demonstrate how this representation of damping changes when going from the elastic through the inelastic ranges, relating the sensitivity of these changes to the variations selected in No. 1.

4. Develop experimental data to verify ductility factors used in conjunction with deterministic and probabilistic analyses.

5. Develop experimental data that will enable others to validate computer programs used to predict the behavior of noncontainment Category I structures in the elastic and inelastic ranges.

The predominate feature of the typical structure under investigation is that shear rather than flexure is dominant, that is the ratio of displacement values calculated from terms identified with shear deformation to the values contributed from bending deformation is one or greater; thus these buildings are called "shear wall" structures.

Results from the Seismic Category I Structures Program through the end of FY84 (September 1984) were described at this conference last year.[1] This paper will describe current program emphasis to determine the credibility of previous experimental work and future program direction.

STIFFNESS DIFFERENCE AND SCALABILITY ISSUE

The experimental program plan was developed with the foreknowledge that scale model testing of reinforced concrete structures is a somewhat controversial issue in the U.S. civil engineering community, particularly when the structures are loaded into the inelastic range. The similitude requirements for our models were carefully considered and discussed in detail in Ref. 2. The experimental plan incorporated both static and seismic testing-to-failure of scale model Category I box-like structures as well as tests on isolated shear walls. The isolated shear wall tests were carried out first; they were then followed by static and seismic tests on one and two-story box-like structures. To verify that the scaling relationships could be used to translate test results to different models and prototypical structures, two 1/30-scale and one 1/10-scale models of a two-story Diesel Generator Building structure were seismically tested. The first 1/30-scale model structure was tested to aid in the development of the test program for the 1/10-scale structure. After the 1/10-scale model tests, the second 1/30-scale model was tested in a manner similar to the 1/10-scale model.

Fig. 1 compares data taken from tests on a 1/30-scale model Diesel Generator Building (3D-12-2) and one 1/10-scale model (CERL No. 2). When the measured first-mode frequency is normalized by the frequency scale factor, N_f, and the peak acceleration is normalized by the acceleration scale factor, $N_{\ddot{y}}$, the data can all be plotted on the same curve. In this notation, the scale factor indicates the ratio of the prototype to the model. In addition, the models had the appropriate added masses, and the base motion was properly frequency scaled so that the 1/30-scale structure is a true 1/3-scale model of the 1/10-scale structure while both structures are models of the assumed prototype. When the data are illustrated as in Fig. 1, the prototype behavior is shown directly, while the individual model data require knowledge of the scale factors (1/30 scale: N_f = 1/11.8, $N_{\ddot{y}}$ = 1/4.6 and 1/10 scale: N_f = 1/6.8, $N_{\ddot{y}}$ = 1/4.6).

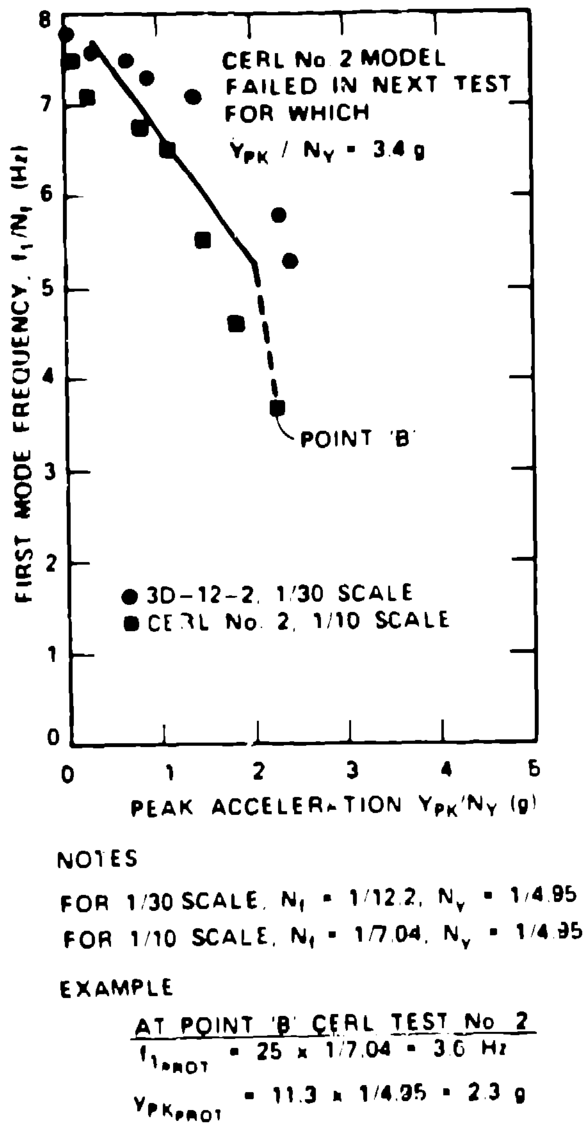

Fig. 1. Data illustrating the first mode frequency shift as the model structures were progressively damaged by increasing peak seismic base accelerations.

Clearly, the scalability of the results from seismic testing the two different sized models is demonstrated, but because both models are made of microconcrete with simulated rebar, scalability to the prototype structure is still an issue. In addition, both static and dynamic tests using isolated shear walls and box-like structures indicate that the stiffness is significantly less than the stiffness computed assuming an uncracked concrete cross section.

The lower than expected initial stiffness is further addressed in Fig. 2. This figure illustrates the secant stiffness plotted against the concrete modulus, E_c. The secant stiffness was taken at 50% of the ultimate load (measured from experimental results) normalized by the structure's theoretical value calculated from an uncracked cross-section strength-of-materials approach. The concrete modulus was obtained from the equation $E_c = 57000 \sqrt{f'_c}$ as recommended in ACI 349 for normal weight concrete. With the exception of a single point (a "wet" test in an aging study) the data consistently show that calculated stiffnesses are down by a factor of 3 to 4 at this load level. Similar differences have been reported in certain papers in the literature. On the other hand, values reduced by 20% or less have also been indicated in the literature.

The point marked "Sozen" was deduced from Ref. 3 and should be explained. The initial stiffness found from a pluck test on the model in Ref. 3 was almost the theoretical value. The point shown on Fig. 2 is the stiffness of the structure as found after subjecting it to a 1/4-g seismic excitation. The point marked "Unemura" was taken from the figures of Ref. 4 using the same method we have used on our data.

Early in the life of this program, a Technical Review Group (TRG) consisting of nationally recognized seismic and concrete experts on nuclear civil structures was established to both review the progress and make recommendations regarding the technical directions of the program. The recommendations of this group have been evaluated in light of the needs of the USNRC and, where possible, have been carefully integrated into the program.

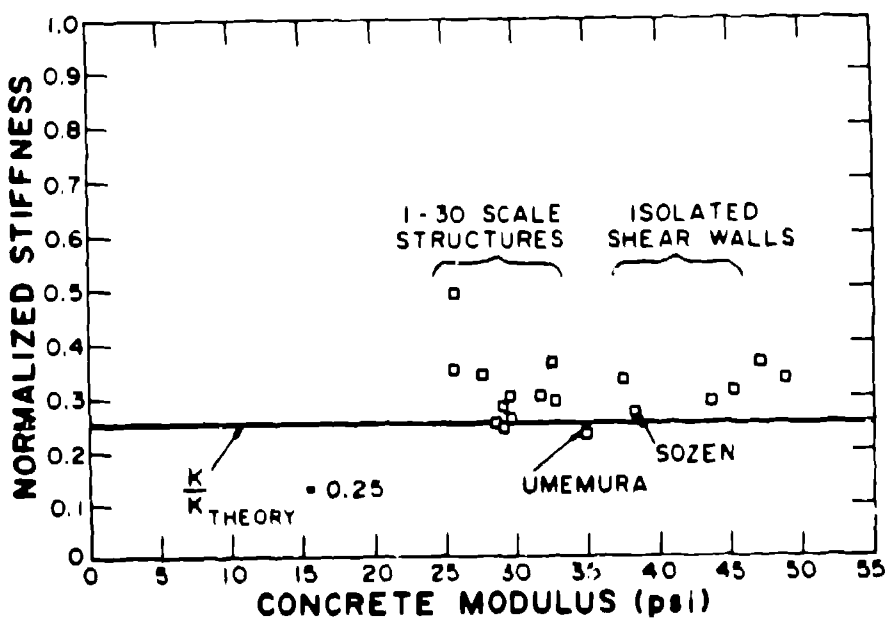

Fig. 2. Normalized stiffnesses versus concrete modulus from this program and other literature values.

During their review of the data through FY84, the TRG pointed out the following:

1. Design of prototype nuclear plant structures is normally based on an uncracked cross-section strength-of-materials approach that may or may not use a "stiffness reduction factor" for the concrete; however if one is used, it is never as large as 4.

2. Although the structures themselves appear to have adequate reserve margin (even if the stiffness is only 25% of the theoretical), any piping and attached equipment will have been designed using incorrect floor response spectra.

3. Given that a nuclear plant structure designed to have a natural frequency of about 30 Hz really has a natural frequency of 15 Hz (corresponding to a reduction in stiffness of 4), and allowing further that the natural frequency will decrease because of degrading stiffness, the natural frequency of the structure may shift well down into the frequency range for which an earthquake's energy content is the largest. This will result in increased amplification in the floor response spectra at lower frequencies, and this fact potentially has significant impact on the equipment and piping design response spectra and equipment and piping margins of safety.

Note that all three points are related to the difference between measured and calculated stiffnesses of these structures.

Having made these observations, several questions now arise. Does our previous experimental data taken on microconcrete models represent data that would be observed on prototype structures? What is the appropriate value of the stiffness that should be used in design and for component response spectra computations in these structures? Should it be a function of load level? Have the equipment and piping in existing buildings been designed to incorrect response spectra?

Thus, the primary program emphasis at this time is to assure credibility of previous experimental work by beginning to resolve the "stiffness difference" issue. The Technical Review Group (TRG) for this program believes that this important issue must be addressed before the program objectives can be accomplished.

To address these stiffness-related concerns, it was agreed that a series of credibility experiments will be carried out using both large and small-scale structures. For the large-scale structure, the TRG set limitations on the design parameters. Their recommended "ideal" structural characteristics, in order of decreasing priority, are as follows:

1. provide a maximum predicted bending and shear-mode natural frequency $\leq$ 30 Hz

2. use a wall thickness $\geq$ 4 in.

3. use a height-to-depth ratio of shear wall $\leq$ 1

4. use actual No. 3 rebar for reinforcing

5. use realistic material for aggregate

6. use 0.1 to 1% steel (0.3% each face, each direction ideally)

7. use water-blasted construction joints to assure good aggregate frictional interlock.

It was further agreed that the best plan would be to build two of these structures as nearly identical as possible. To compare the results from these tests with previously obtained data, one model should be tested quasistatically and cyclically to failure, and the second model should be test dynamically.

Following these recommendations and other TRG suggestions, and after analyzing a number of potential designs, the structure shown in Fig. 3 was proposed

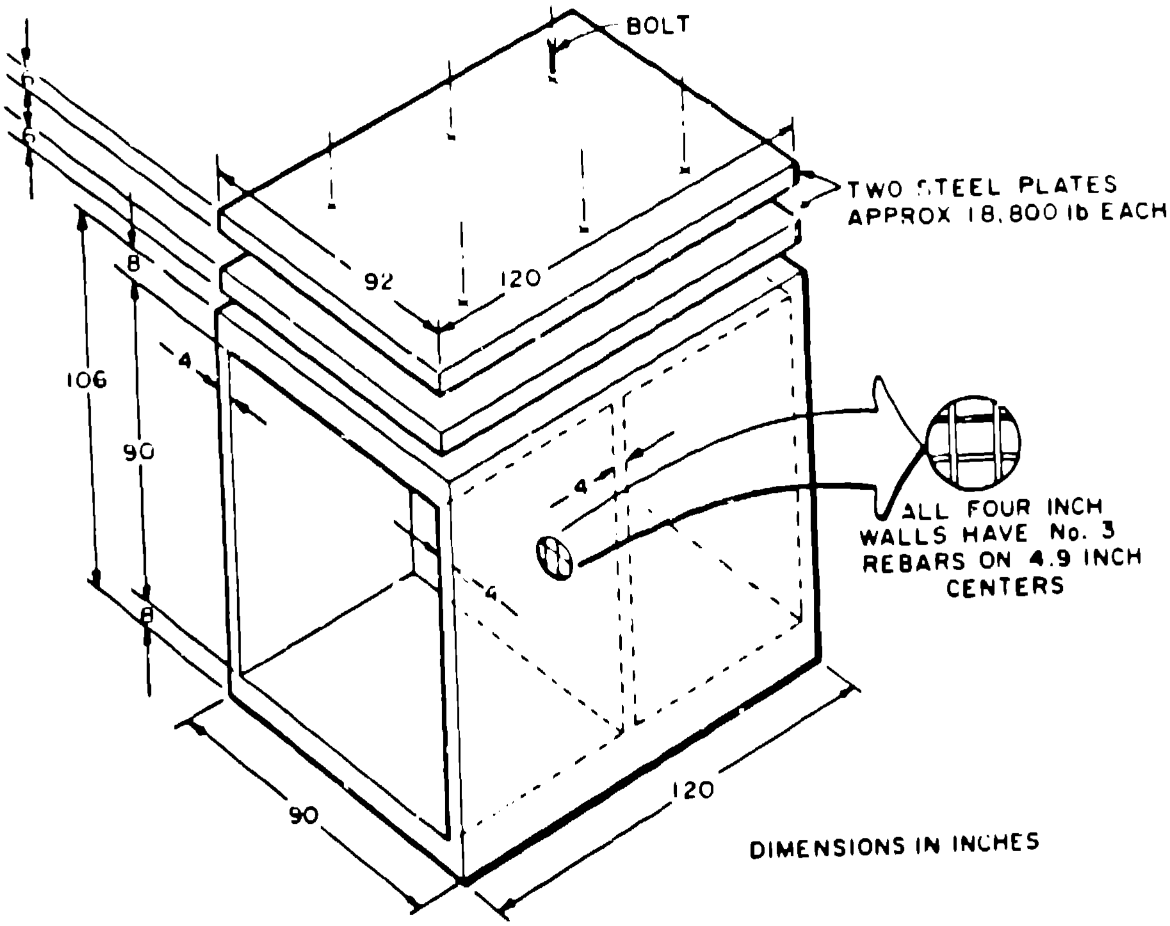

Fig. 3. TRG structural test model.

to both the TRG and NRC as being a test structure fulfilling the design requirements. Table I gives some of the details of this structure. After resolving a number of questions relating to the details and the potential response (dealing with out of plane bending of walls, torsion, etc.,) of the structure, the decision was made to construct and test this particular configuration and its models.

TABLE I
COMPUTED CHARACTERISTICS OF THE TRG MODEL STRUCTURE

$I_{\text{uncracked transformed section}}$	= 2.06×10^6 in.^{4}
A-effective shear	= 379 in.^{2}
Area total	= 1288 in.^{2}
Total uncracked bending stiffness	= 2.5×10^7 lb/in.
Shear stiffness	= 5.3×10^5 lb/in.
Total stiffness	= 4.2×10^6 lb/in.
Max dead weight normal stress	= 42 psi
Max shear stress in flange at 5 g due to assumed 5% torsion (approx.)	= 35 psi
Total concrete	6 cubic yards
Total added weight	37,000 lb.
Total weight	61,000 lb.

Treating the TRG structure of Fig. 3 as the "prototype," the plan was to first construct 1/4-scale Case-I type models from microconcrete. In a Case I model, the mass is scaled by the length scale cubed. All gravitational effects are distorted (they are too low) by a factor of the length scale. For example, normal dead weight stresses are 10 psi in a 1/4-scale model instead of 40 psi, but both values are small compared to the cracking strength of the concrete. Overturning moment due to gravity is low by 4, but the overturning moment due to the inertia force is scaled correctly (and is usually orders of magnitude larger than that due to gravity alone.) In general, for this model as with the other models used in this program, the magnitude of the distortions and their effects are understood and are deemed to be acceptable. The major exception is the scaling effects associated with the use of microconcrete.

THE ONE-QUARTER-SCALE MODELS

The purpose of the 1/4-scale models is as follows: first, by applying the same principles of analysis and design and construction practices as have been applied in our previous work, we will attempt to demonstrate the scalability of the results to the prototype TRG model. Second, conclusions (based on calculations) concerning the model and prototype torsional response, individual wall frequencies, out-of-plane bending, and other features that affect the response of the large TRG structure can be confirmed on an inexpensive test structure. Third, instrumentation and other data acquisition requirements can be worked out in advance of the larger scale tests. As an example, a good analytical model may have to treat the shear stiffness before and after cracking quite differently, and instrumentation to separate overall shear distortion from overall bending distortion on the large model in the static loading case has been proposed. This instrumentation has been designed and checked on the small model. Also, low load-level testing (modal and static) is a new feature for this program, which in the past has been concerned with working load levels. Methods and details for this type of testing have been worked out using the small models.

The two 1/4-scale models have been completed, and testing of the second model is in progress with testing of the first being complete.

The 1/4-scale models were constructed of microconcrete using our previous construction experience. A double row of 1/4-inch hail screen reinforcing simulating 0.56% steel in each direction was placed on the centerline of each end wall and the shear wall. The top and bottom slabs were heavily reinforced with No. 3 bars. Properties of the first model's reinforcing and the microconcrete are given in Table II.

TABLE II
MATERIAL PROPERTIES FOR TRG MODEL I

Concrete

E	=	(measured at $\sigma - \epsilon$ origin) = 3.18×10^6 psi
f'_c	=	(compressive strength) = 3769 psi
f_t	=	(split tensile test strength) = 513 psi
E'_c	=	$57000 \sqrt{f'_c} = 3.5 \times 10^6$ psi

Steel - Bilinear Properties - 0.6% Both Directions

E	=	25.6×10^6 psi
Yield Strength	=	42.7 KSI
Ultimate Strength	=	53.1 KSI
Elongation at Failure	=	0.04
Diameter	=	0.042 in.

TESTING PROGRAM

The testing program for this model consisted of a series of very low load-level modal and static tests followed by increasingly severe random and simulated seismic testing to failure. The low load-level testing were all "bare" model tests (no added mass), and the random and seismic tests were conducted with 575 lb. of added weight as is appropriate for a 1/4-scale model of the large 30-Hz TRG structure.

DETERMINATION OF INITIAL STIFFNESS

The primary purpose of all low-level tests was to compare the so-called "undamaged" stiffness or virgin model stiffness to the theoretical values. A model shear-bending stiffness was deduced from all modal and low-level static tests, and these values are given below in Table III. The consistency of the values between the static (direct measurement) and dynamic (indirect measurement) methods is obviously good.

Table IV presents the results of all calculated values using both the strength-of-materials approach and a finite element calculation, and the three various estimates for the concrete modulus, $E_c = 3 \times 10^6$ psi (design value), $E_c = 3.18 \times 10^6$ psi (strain-gage measured value), and $E_c = 3.5 \times 10^6$ psi (ACI Method, $E_c = 57000\sqrt{f'_c}$). Clearly, the measured values of the stiffness at low levels are within 70-90% of theoretical values.

WORKING LOAD LEVEL TESTS

Following the low-load level testing, the model was subjected to a random and seismic load test plan similar to the test plan used to test all previous models. First, bare model tests were carried out with 0.5-g random base excitation followed by a seismic input that varied from 0.5-g nominal to 1-g nominal. These bare model tests were used to characterize the "undamaged" stiffness at a higher load level than those used in the modal and low-level static tests. These tests indicated a reduction in stiffness over the low-load level value of about 24%. Next, weights (575 lb) were added to the model to fulfill similitude requirements for a 1/4-scale model of the large TRG structure. The initial tests in this configuration were used to calculated the working load stiffness as in previous models and indicated a stiffness of 441200 lb/in, approximately 38% of the value that would be calculated by an uncracked strength-of-materials approach. This value is consistent with values reported in all of our previous tests on the 3-D structures. Figure 4 illustrates this point which shows the normalized measured stiffness reported from previous tests and this current model test structure, when it was subjected to the same testing procedure as in the previous tests on the 3-D test structures.

TABLE III
MEASURED VALUES OF INITIAL STIFFNESS

	Stiffness $\times 10^6$ lb/in
<u>Static or Direct Measurements</u>	
Dial gage data	0.915
Noncontact gage data	0.695
All static data	0.752
<u>Dynamic or Indirect Measurements</u>	
Free-free modal Test 1	0.775
Free-free modal Test 2	0.707
Fixed-free modal test	0.802
Average value from all data	0.774

TABLE IV
CALCULATED VALUES OF STIFFNESS

Method and Assumptions	Stiffness $\times 10^6$ psi
Strength-of-Materials Approach	
$E_c = 3.00 \times 10^6$ psi	1.09
$E_c = 3.18 \times 10^6$ psi	1.15
$E_c = 3.50 \times 10^6$ psi	1.27
Finite Element Method	
$E_c = 3.00 \times 10^6$ psi	0.860
$E_c = 3.18 \times 10^6$ psi	0.910
$E_c = 3.50 \times 10^6$ psi	1.00

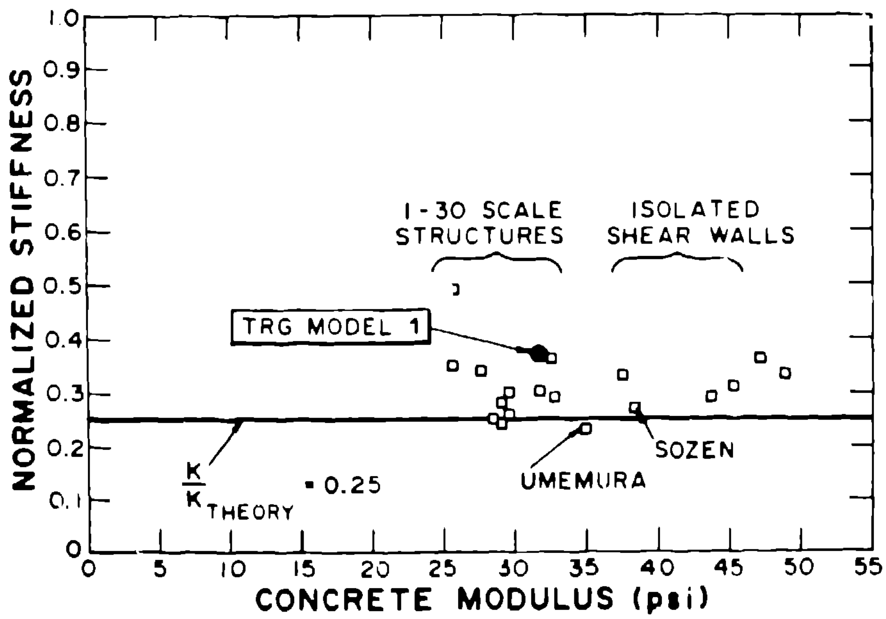

Fig. 4. Normalized stiffnesses versus concrete modulus from this program and others, and showing the 1/4 scale TRG model after being subjected to 1/2 g seismic test.

To date, we believe the TRG series of tests will be valuable in resolving the modeling and reduced stiffness issue. The most important tests in this regard will be the tests of the large models, which are scheduled to begin in November and December.

FUTURE EXPERIMENTS

Following the resolution of the stiffness difference issue, a limited number of tests will be carried out to meet program objectives and aid in benchmarking the analytical model development. If settlement of the scalability and stiffness difference issues allows, these tests will be carried out on one-inch-thick wall concrete models. A statistician, knowledgeable in experimental design, will be used to comment on the test configurations recommended and to assure that the controlled variables (i.e., number of floors, wall arrangement, etc.) and uncontrolled variables (i.e., concrete strength) are incorporated into a cost-effective test matrix to meet program objectives.

One further effort will be investigated and possibly initiated. The program management has noted instances in the shock and vibration literature of researchers measuring and reporting natural frequencies and mode shapes of very large reinforced concrete structures. This developing technology will be investigated and the possibility of performing such a test on a full-scale Category I shear wall structure evaluated. If a suitable structure can be found, such a test combined with analytical modeling could be used as a final confirmation of the as-built stiffness of prototype structures.

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